

A Summary of Test Results for Selective Weakening and Post-tensioning for Retrofit of Non-Ductile R.C. Exterior Beam-Column Joints

W.Y. Kam, S. Pampanin

University of Canterbury, Christchurch

D. Bull

Holmes Consulting & University of Canterbury, Christchurch



2010 NZSEE
Conference

ABSTRACT: This paper summarises the experimental results of a counter-intuitive seismic retrofit strategy – Selective Weakening, targeted at improving the seismic performance of poorly detailed (pre-1970) exterior beam-column joints, a critical weakness of non-ductile reinforced concrete frames. The retrofit interventions investigated comprised of: a) selectively weakening the beams (reducing joint shear demand) and/or b) adding external post-tensioning to the beam-column joints (adding horizontal confinement and axial force to the joint) with the aim to change the local inelastic mechanism of the external joints. This would result in improved global displacement, ductility and energy dissipation capacities without significant increase in demand of foundation strength, typical of strengthening retrofit. Extending from previously presented preliminary experimental results, a comprehensive test matrix and its results are presented herein. Nine 2/3-scaled exterior beam-column joints subassemblies – as-built and retrofitted were constructed and tested to investigate the feasibility and effectiveness of the proposed retrofit strategy. Parameters considered in the tests were the levels of external post-tensioning forces, locations of beam weakening, the presence of column lap-splice and the presence of cast-in-situ slab and transverse beams. Experimental results have indicated that joint shear and column lap-splice failures can be prevented or mitigated using a Selective Weakening retrofit approach, leading to the targeted seismic performance. This research is part of the six-year NZ FRST national project on “Seismic retrofit solutions for NZ multi-storey building”.

Keyword: Selective weakening, seismic retrofit / rehabilitation, beam-column joint, reinforced concrete, external post-tensioning

1 INTRODUCTION

Poorly detailed, pre-1970s beam-column (b-c) joints are identified as a critical weakness of non-ductile reinforced concrete (RC) frames. Laboratory tests of non-ductile RC b-c joints [12] as well as field observations in recent earthquakes [3, 13] have further confirmed the seismic vulnerability of these pre-1970s b-c joints. Some examples of structural and soft-storey collapses, initiated from b-c joint failures are presented in Figure 1.

In resolving this seismic deficiency of pre-1970s RC frames, various seismic rehabilitation solutions have been previously proposed and implemented for b-c joints[1, 2]. The majority of the established methods involve either the strengthening of the joint-only or both the joint and column in order to induce plastic hinging in the beams. Alternatively, the demand on the structure can be reduced by supplementary damping or base-isolation.

In some previous contributions at this conference, the concept of Selective Weakening (SW) retrofit of RC structures has been introduced [11] and demonstrated for the retrofit of shear-deficient structural walls [4]. The authors [7] have also presented in the last conference the extension of SW retrofit for non-ductile RC frames and the preliminary experimental results of four 2/3-scaled exterior b-c joints as a proof of concept. Since, further numerical and experimental works were done to further quantify

the seismic performance of SW-retrofitted exterior b-c joints under various retrofit design and as-built parameters[5, 6, 8, 9].



Figure 1: Structural collapse and damage due to b-c joint failures a) Kaiser-Permanente Clinic, Northridge 1994 [3], b) Yingxiu school, Wenchuan, China 2008 [13] c) Femina Hotel, Padang, Indonesia 2009 [10].

SW retrofit aims to improve global inelastic mechanism and the deformation capacity of the RC frame, by first weakening and then upgrading specific /critical structural (or non-structural) elements, as illustrated in Figure 2. Particularly, to induce a flexural hinge in the beams, some (or all) longitudinal beam reinforcements are cut at the exterior b-c joint faces. The overall frame, whilst weakened, achieves higher deformation and ductility capacity. Further strengthening with external post-tensioning can improve the lateral capacity and energy dissipation, while still respecting the capacities limit of other critical elements (e.g. columns and foundation).

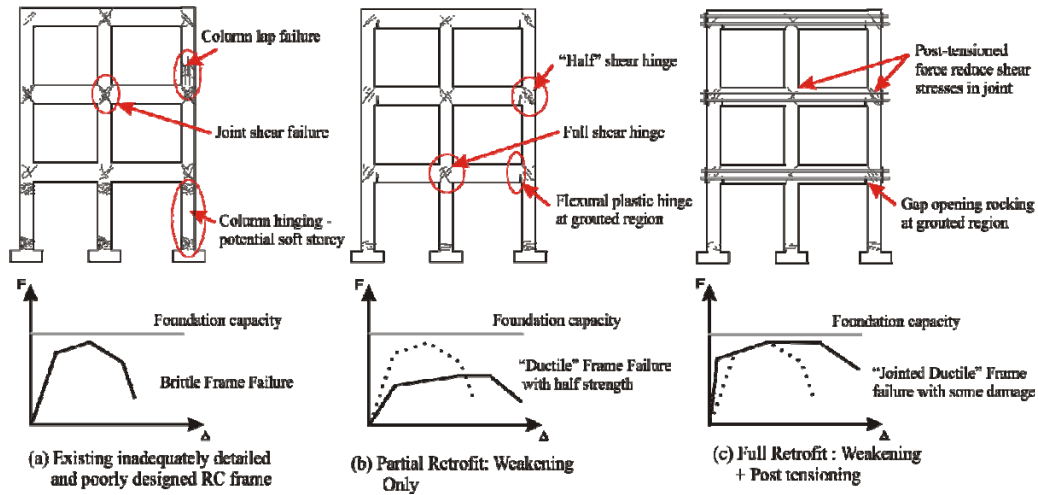


Figure 2: SW retrofit for RC frame: a) Non-ductile RC frame b) Beam weakening-only retrofit c) Beam weakening and external post-tensioning of joint retrofit.

2 EXPERIMENTAL PROGRAMME

2.1 Test Matrix

Nine 2/3-scaled exterior b-c joints subassemblies – three as-built and six retrofitted specimens were constructed and tested under quasi-static uni-directional cyclic loading. The prototype b-c joint was designed to represent the worst typical-case in pre-1970s RC construction practice. The joint core had no transverse reinforcement and the beams longitudinal reinforcements were anchored using 180° standard hooks into the joint. Figure 3 shows the prototype RC frame and the reinforcing details adopted for the experiment. In the 2/3-scaled models, all test units had 230mm x 230mm (9x9 inch) columns and 330mm deep x 230mm wide (13x9 inch) beams. Standard steel products were used: mild steel and pre-stressing 7-wire 12.7mm diameter strands with yield strength of 330MPa (47.9ksi) and 1560MPa (226.3ksi) respectively. Other materials and reinforcing details are as given in Table 1.

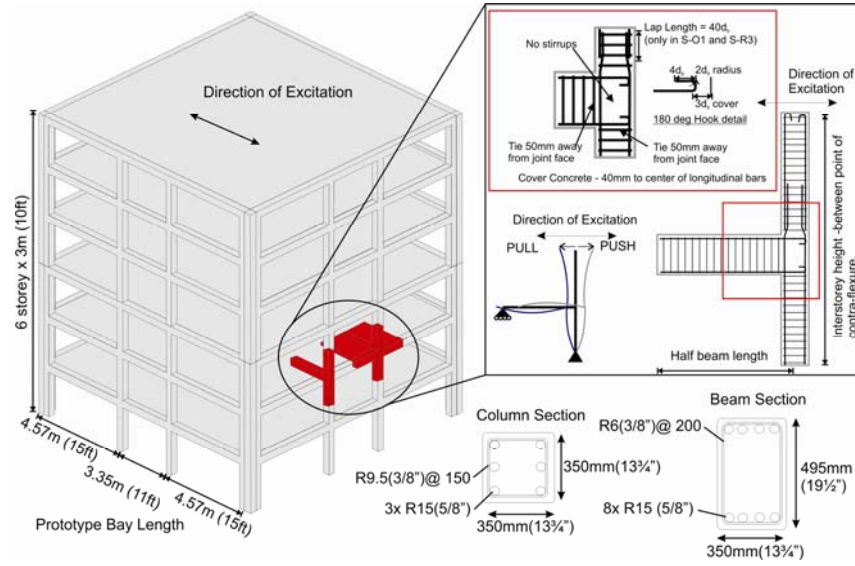


Figure 3: Prototype RC frame and non-ductile detailing and geometry adopted for prototype beam-column joints.

Parameters considered in the tests included the levels of external post-tensioning forces, locations of beam weakening, the presence of column lap-splice and the presence of cast-in-situ slab and transverse beams. The description of the test units are given in Table 1. Three as-built b-c joints were used to benchmark seismic performance of un-retrofitted joints, labelled with the suffix -O1. The prefixes of NS-, S- and SL- represent the three different configuration of as-built benchmark: a) NS-: plane b-c joints with standard details as shown in Figure 4a, b) S-: as-built b-c joints with deficient column lap-splice and c) SL-: as-built b-c joints with floor-slab and transverse beams.

Table 1: Description of beam-column joint test units

Test Unit	Description	Beam Bottom Reinforcement	Cutting Radius (mm)	Weakened section distance from column C/L (mm)	Post-tensioning Force (kN)	Concrete Strength, f_c (MPa) ¹	Grout Strength, f_g (MPa) ²
NS-O1	as-built benchmark specimen	4-R10	-	-	-	17.3	-
S-O1	as-built specimen with column lap splice	4-R10	-	-	-	15.1	-
SL-O1	as-built specimen with slab/transverse stub	4-R10	-	-	-	13.4 & 19.9 ³	-
NS-R1	R1 Retrofit: beam-weakening only	2-R10 ⁴	80	165	-	25.6	30
NS-R2	R2 retrofit: external post-tensioned (PT) only	4-R10	-	-	120	28.2	-
NS-R3	R3 Retrofit: beam weakening and external PT	2-R10 ⁴	80	165	40	24.3	30
NS-R4	R4 Retrofit: beam weakening and external PT	2-R10 ⁴	80	310	24	30.3	-
S-R3	S-O1 specimen with R3 retrofit scheme	2-R10 ⁴	80	165	40	20.7	30
SL-R3	SL-O1 specimen with R3 retrofit scheme	2-R10 ⁴	80	165	40	17.0 & 23.1 ³	30

Abbreviation: NS=no column lap-splice; O=as-built; R=retrofitted; PT=post-tensioning; R10 = plain round bars with diameter 10mm.

¹ Concrete strength at the day of testing; ² Specified strength from manufacturer at testing day; ³ Top half of the column and other parts were casted separately.

The first value given is the top half of the column concrete strength. ⁴ Selective beam weakening with two outer bottom longitudinal bars severed.

The four retrofit solutions were labelled with the suffix R1, R2, R3 or R4. The R1 scheme was a Partial SW solution, where 50% of the bottom longitudinal beam bars were cut. Aiming at reducing the joint shear demand as well as inducing flexural hinge in the beam, the R1 retrofit scheme was designed as a collapse-prevention retrofit intervention. The R2 solution was also a Partial SW solution, where the joint and beam was post-tensioned with two 60kN externally anchored tendons. The increased axial stress in the joint and the confinement effect improved the joint shear behaviour significantly while the post-tensioning increased the flexural and shear capacities of the beam. In the design, the specimen NS-R2 was expected to hinge in the beam and column, depending on the variation of axial load. This gave a further rationale to combine the Partial SW solutions to achieve a higher performance level. The R3 and R4 solutions were Full SW retrofit schemes, where the beams were selectively weakened in conjunction with external pre-stressing of the b-c joint. The locations of

beam weakening and levels of post-tensioning force were varied between the two solutions R3 and R4.

2.2 Testing Setup and Loading Protocol

To simulate earthquake loading, cyclic quasi-static lateral loading was applied horizontally at the top of the column, as shown in the test setup (Figure 4a). The loading protocol used consisted of two displacement-controlled cycles at increasing amplitudes as follows: 0.1%, 0.2%, 0.5%, 1.0%, 1.5%, 2.0%, 2.5%, 3.0% and 4.0% inter-storey drifts (Figure 4b). Varying axial load of $120\text{kN} \pm 4.63F_C$ was implemented, where F_C was the lateral force applied to the top of the column. The varying axial load ratio (4.63) was based on the prototype RC frame shown in Figure 3. For specimens with slab, realistic gravity loadings were added. All the specimens were thoroughly instrumented to measure: a) lateral force applied b) column displacements c) local deformation components, and d) strains in the steel reinforcement.

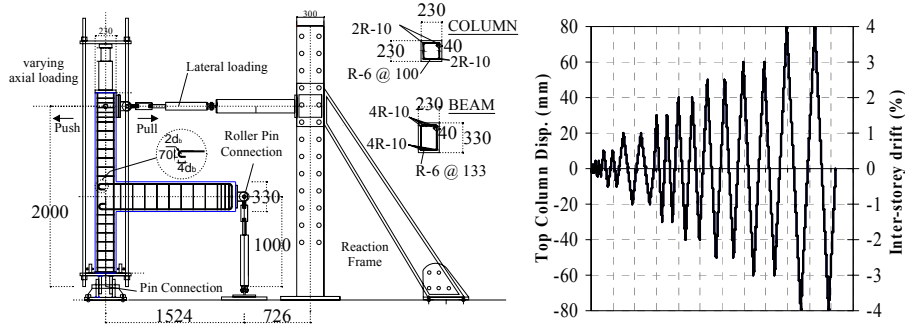


Figure 4: a) Experimental test setup (unit shown are in mm) b) Loading protocol

3 TEST RESULT SUMMARY

The experimental results are summarized in Table 2 and the associated hysteresis curves of all b-c joints are presented in Figure 5. The final damage patterns of the test units are given in Figure 6. All b-c joints were tested up to 4.0% cycles except for NS-O1, which failed prematurely at the end of the 2nd cycle at 3.0% lateral drift. It is noted that the ‘ultimate’ failure point is defined as the deformation corresponding to the reduction of 20% of the original peak force within the same drift cycle.

Table 2: Summary of experimental test results

Test Unit	Failure Mode	Peak Lateral Force (kN)	Inter-storey drift at maximum force, θ (%)	Ultimate inter-storey drift, θ (rad) ¹	Inter-storey drift at Joint Cracking, θ (rad) ²
NS-O1	Joint Shear Failure	+14.7 -18.7	+1.95 -0.93	+1.0%-II	+0.9-I
S-O1	Joint Shear & Lap Splice Failure	+14.1 -16.7	+1.43 -0.98	+2.5%-I	+0.7-I
SL-O1	Partial-confined Joint Shear	+21.2 -16.3	+2.42 -2.45	-3.0%-II	+1.0-I
NS-R1	Beam Flexural, Anchorage	+8.2 -15.4	+0.95 -0.80	-2.5%-II	na ³
NS-R2	Beam and Column Hinging	+18.4 -25.2	+3.56 -1.96	-4.0%-II	+1.5-I
NS-R3	Beam Flexural Hinging	+17.4 -21.6	+3.93 -3.91	na ³	-4.0-I
NS-R4	Beam Flexural Hinging	+14.9 -22.6	± 4.0	na ³	+2.0-I
S-R3	Beam Flexural Hinging	+15.9 -21.5	± 4.0	na ³	-2.0-I
SL-R3	Beam and Column Hinging	+21.3 -29.3	+3.94 -2.95	na ³	na ³

Positive force, displacement and drift correspond to PULL cycles while negative values indicate PUSH cycles. I=1st cycle; II=2nd cycle

¹ Failure point defined as attained peak force was less than 80% of previous peak force; ² Joint cracking was observed as the appearance of diagonal shear crack and/or sudden drop of lateral load due shear cracking. ³ No failure/cracking (based on the definition) achieved.

For NS-O1, joint shear failure – with subsequent concrete wedge spalling and column bars buckling, governed its premature failure mechanism. For S-O1, the column lap-splice while limited the joint shear stress demand, accelerated column longitudinal bars buckling, cyclic strength degradation and premature failure. With floor slab and transverse beams, positive effects on the post-joint-cracking behaviour of SL-O1 were observed. Despite early joint cracking (1st cycle of 1.0% drift) and

predominantly shear-hinging inelastic mechanism, SL-O1 activated a relatively stable, despite thin hysteresis loop.

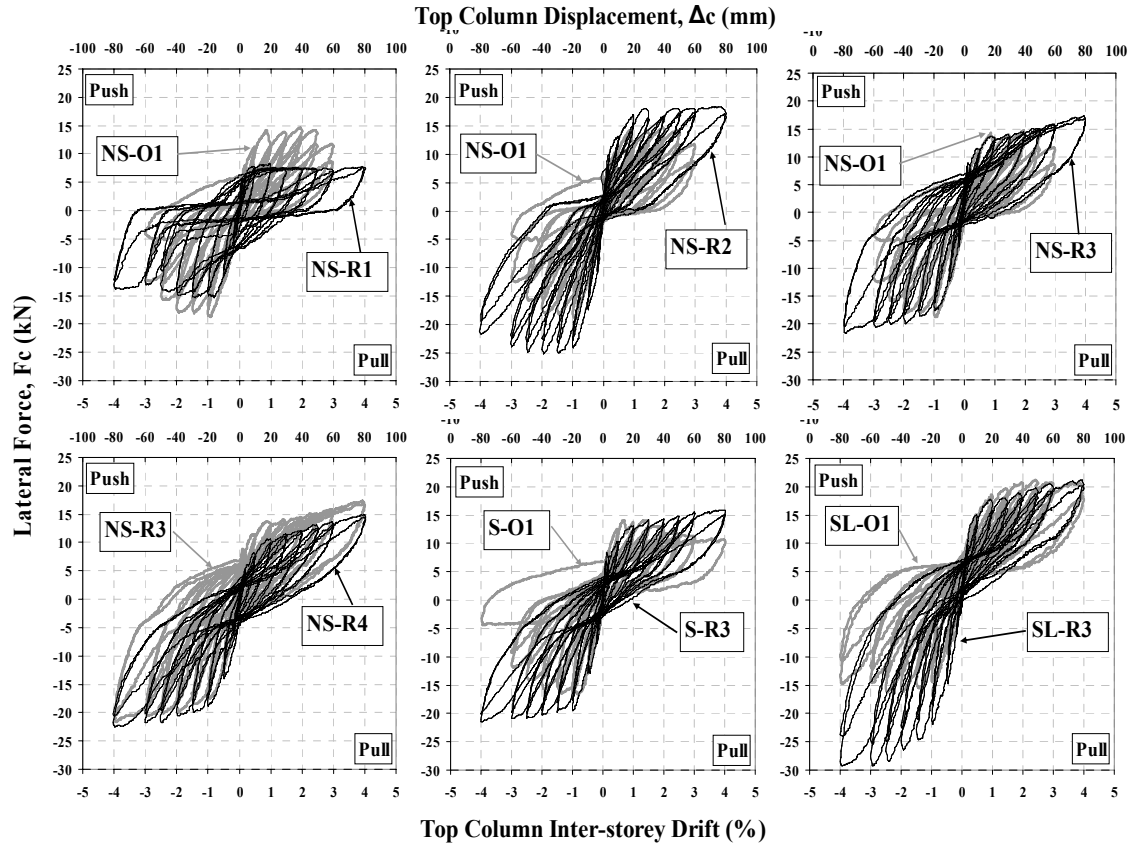


Figure 5: Force-displacement hysteresis curves.



Figure 6: Failure and damage patterns at the end of the tests.

The two intermediate retrofit solutions – beam-weakening-only, NS-R1, and external post-tensioning-only, NS-R2, demonstrated the possibility of simple collapse-prevention retrofit. NS-R1 retrofit was successful up till 2.5% inter-storey drift before failing (rapidly) in compression anchorage push-out. Considering the large hysteresis loops prior to failure, a non-ductile RC frame with such connection would most probably survive collapse prevention or life-safety limit states[5].

As for NS-R2, partly rocking b-c joint behaviour was attained but limited energy dissipation was achieved due to plain-round bars slipping. Column yielding and hinging beyond 2.5% exacerbated the overall behaviour. The column hinging was activated by the increasing post-tensioning contribution due to gap/crack opening on the b-c interface. Thus, external joint pre-stressing-only solution, as in NS-R2, while prevented joint shear failure, might over-strengthen the beam and result in to column failure, which is undesirable.

As expected, the R3 and R4 solutions gave the most satisfactory performance. The beam weakening reduced the joint shear demand and the joint post-tensioning added confinement and beneficial axial stresses. External pre-stressing avoided anchorage failure as observed in NS-R1. In NS-R3, deformation and energy dissipation capacities were improved without significant over-strengthening of the b-c joint when compared with NS-O1. The weakening and post-tensioning also limited the joint principal-stresses and joint shear deformation below the threshold of joint diagonal cracking at approximately $p^*t=0.2\sqrt{f_c}$ MPa and $\gamma_j=0.5$ rad.

4 DISCUSSION

4.1 Levels of Post-Tensioning Force

Three levels of post-tensioning forces were applied (R2: 120kN, R3: 40kN and R4:24kN), with NS-R2 without any beam weakening. External post-tensioning forces affected the extent of beam strengthening and joint strengthening. As all three specimens had beam hinging up to 1.5% drift, after which NS-R2 started having column hinging, the influence of increased level of post-tensioning forces on the beam flexural strength could be identified. Excessive beam-strengthening would lead to column hinging as in NS-R2 for un-bonded post-tensioning, since it would have high post-yield stiffness. At high drifts, the post-tensioning tendon's moment contribution increased due to larger level arm and higher force from tendon elongation. Another drawback of excessive post-tensioning forces was the possibility of diagonal joint compressive failure, which also occurred for NS-R2 at 1.5% drift cycles.

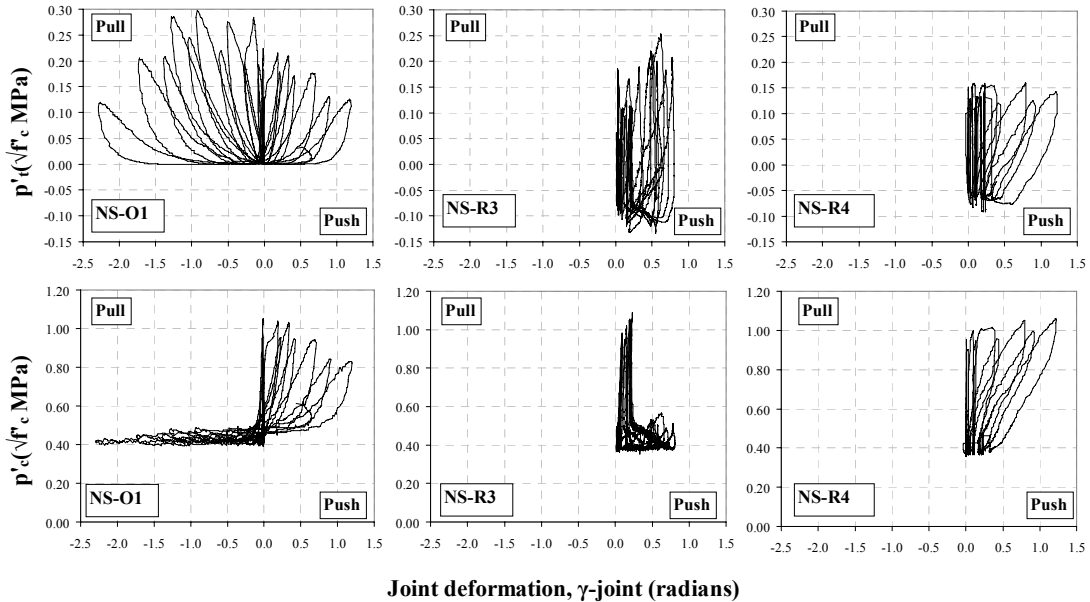


Figure 7: Joint shear deformation and joint principal stresses for NS-O1, NS-R3 and NS-R4.

The lower post-tensioning forces led to lower joint shear resistance in the NS-R4. Diagonal joint tensile cracking occurred at 2.0% pull-push drift cycles, when compared with joint tensile cracking at 4.0% drift for NS-R3. Globally, this led to lower energy dissipation and more cyclic strength degradation, when comparing NS-R4 with NS-R3. Evaluating Figure 7, it can be seen that for the as-built NS-O1, the joint failed in principal tensile stress in the Pull direction while it failed in principal compressive stress in the Push direction. Comparing NS-R3 with NR-R4, despite both specimens having roughly the same maximum principal stresses, NS-R4 had more extensive joint damage and deformations in the Push direction. This can be attributed to the lower post-tensioning force in the R4 solution.

4.2 Locations of Beam Weakening

The shift in beam-weakening location in the NS-R4 specimen was targeted to relocate the plastic hinge in the beam away from the joint. This would further reduce joint shear demand and give a longer anchorage length to the beam longitudinal reinforcements, assuming the weakest section would be at the weakened section. From Figure 5, it is evident that NS-R3 had higher lateral force capacity (F_c) in the Pull direction than NS-R4, while both specimens had similar F_c in the Push direction. The decreased F_c for NS-R4 can be attributed to a) the shifted negative moment flexural hinge to the weakened section, and b) lower post-tensioning force as discussed in the previous section.

To investigate whether bonds of the beam reinforcements have better capacity in NS-R4, the strain gauge readings for the top and bottom (uncut bars) during the 1st Pull peak cycles were analysed and presented in Figure 8. NS-R4 developed higher steel strains/stresses in both top and bottom reinforcements compared with NS-R3— alluding to the improved anchorage and bond performance of the NS-R4. Bond slip failure occurred in bottom reinforcements during the Push direction, near the b-c face, as the steel strain decreased beyond 1.5% drift cycles. Similarly in Pull 1 cycles, steel strain decreased after 1.5% drift in both specimens. However, it is believed for NS-R3 it was due to bond-slip failure of the bottom reinforcements, while for NS-R4 the strain decrease was due to drop of shear capacity in the joint (after diagonal tensile cracking at 1.5% drift). NS-R4 also clearly illustrated the repeatability of the performance of NS-R3, albeit with lower post-tensioning forces and shifted weakened section.

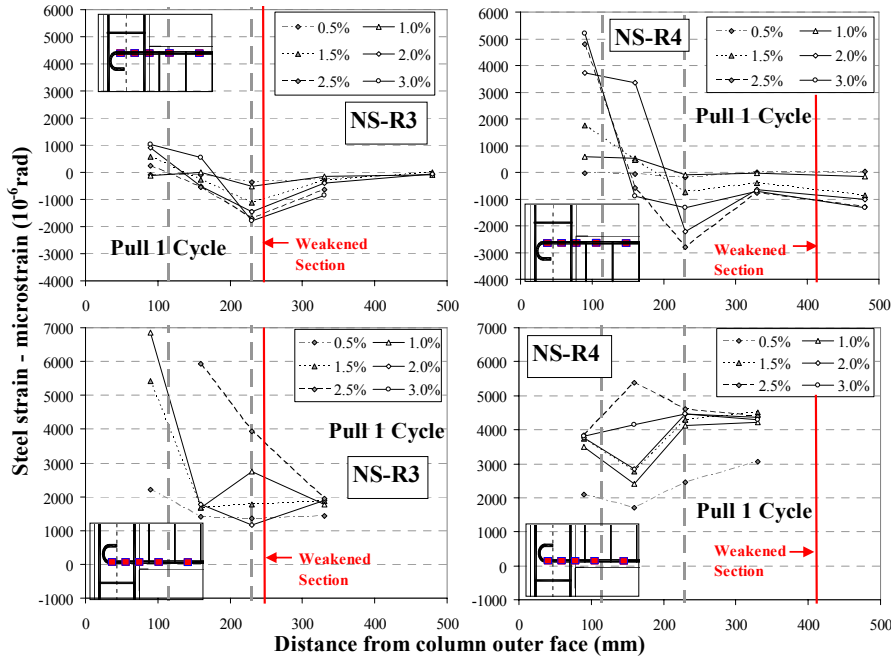


Figure 8: Strain gauge readings for top and bottom longitudinal beam reinforcements for NS-R3 and NS-R4 at 1st peak at Pull direction for different drift cycles. Grey lines are locations for centre-line and inner column faces.

4.3 Influence of Column Lap-Splice

Another limiting factor needed to be considered in the assessment/design is the maximum column moment due to column lap-splice capacity. In terms of as-built b-c joints, NS-O1 and S-O1 both had similar seismic performance characteristics, in which joint shear failure led ultimately to column longitudinal bars buckling and ‘structural collapse’. Figure 9a shows strain gauge data evidence for the lap-splice failure in S-O1. While the retrofitted b-c joint S-R3 prevented strength degradation due to joint shear cracking at 2.0% drift cycles, the beam-weakening was not sufficient to completely protect the column from lap-splice failure. Figure 9b indicates partial lap-splice failure in the exterior column bars. This was further supported by lower global post-yield stiffness and vertical cracking the column for S-R3. Nevertheless, with beam-weakening, the detrimental effect of column lap-splice can be minimised. Further mitigation like local jacketing might be necessary if the columns are expected to yield in the post-retrofitted frame (like base columns).

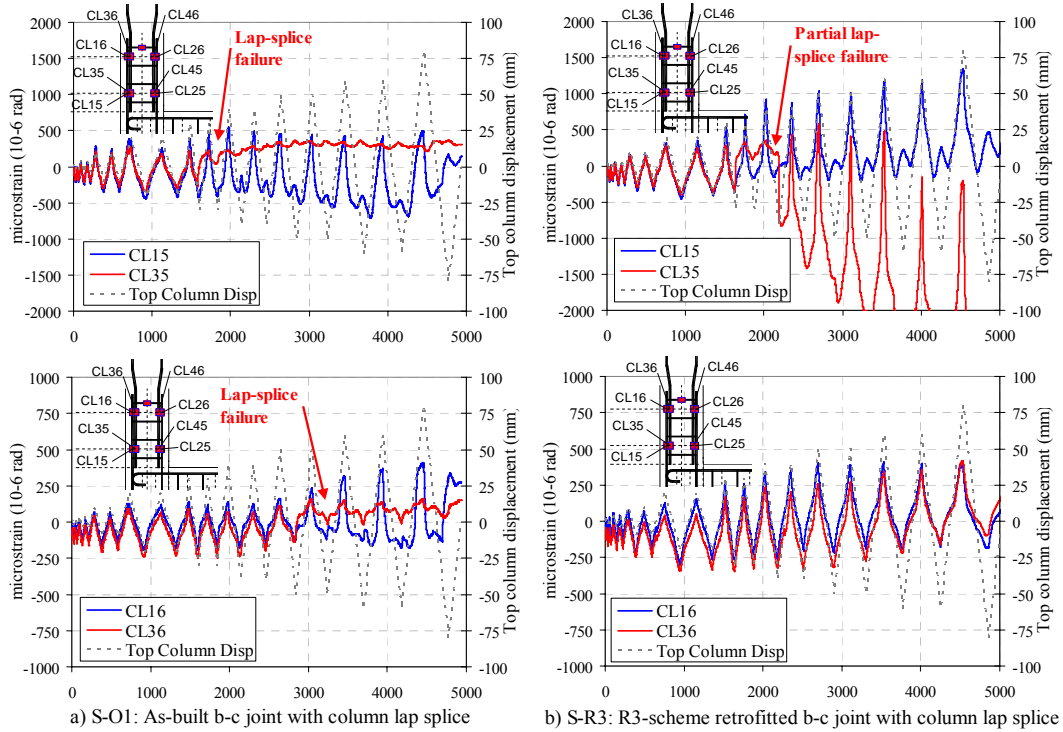


Figure 9: Strain gauge readings for lapped column longitudinal reinforcing bars for S-O1 and S-R3.

4.4 Beam-column Joints with Floor Slab and Transverse Beams

Comparison between NS-O1 and SL-O1 shows the effect of the floor slab and transverse beams in delaying the strength degradation beyond joint cracking. For SL-O1, added displacement ductility without increase in lateral strength or joint shear capacity was observed. From Figure 5, it can be observed that the joint region underwent significant shear deformation in SL-O1. Post-experiment forensic inspection revealed similar diagonal shear cracks within the joint region. The apparent ductility of SL-O1 came from the added confinement from the torsional capacity of the transverse beams, based on the observed 54° inclination torsion cracks on the transverse beams. Joint cracking and bond slip nevertheless resulted in pinching hysteresis for SL-O1. Kam et al [9] gives a more comprehensive look on the influence of floor slab and transverse beams on pre-1970s as-built uni-directional and bi-directional b-c joints

For the retrofitted b-c joint SL-R3, the presence of floor slab and transverse beams increased by nearly 30% the negative moment of the beam and the F_c of the b-c joint when compared with NS-R3. The overall seismic performance was comparable in other aspects. SL-R3 indicated again the importance

of considering the flange-slab effect in assessing the beam flexural capacity, in order to achieve the proper hierarchy of strength within the b-c joints. SL-R3 specimen also demonstrated the practicality and constructability of the retrofit scheme R3 for realistic b-c joint subassembly and under realistic shear demand at the weakened beam section.

5 CONCLUSIONS

This paper has presented further experimental confirmation for a novel retrofit strategy for non-ductile RC frames. By a) selectively weakening the beam of exterior joints (NS-R1), b) upgrading the b-c joints using external pre-stressing (NS-R2) or both a) and b) (NS-R3, NS-R4, S-R3, SL-R3), the joint panel zones were protected and an improved inelastic mechanism was activated. In the comparison of the benchmark b-c joints with various as-built configurations (NS-O1, S-O1, SL-O1), test results indicated the effectiveness of adopting the Selective-Weakening strategy for the retrofit of b-c joints. Being economical, non-invasive and low-technology intensity, it is envisioned that Selective-Weakening retrofit could have a wide implementation potential in a macro-scale retrofit scheme.

ACKNOWLEDGEMENTS:

NZ FRST is acknowledged for its funding of the project “Retrofit Solutions for NZ” (FRST Contract UOAX0411). More information is available at www.retrofitsolutions.org.nz. Special thanks to Mr Mosese Fifita, FRST Retrofit project technician, who assisted in the construction and testing of the specimens.

REFERENCES:

- [1].ASCE-SEI-41-06. Seismic rehabilitation of existing buildings. Reston, Va.: ASCE/SEI; 2007.
- [2].fib. *Seismic assessment and retrofit of reinforced concrete buildings: State-of-the-art report, fib Bulletin no. 24*. Lausanne, Switzerland: International Federation for Structural Concrete (fib); 2003.
- [3].Hall JF. Northridge Earthquake Reconnaissance Report. Supplement C, vol. 1. *Earthquake Spectra*. 1996(11):1-523.
- [4].Ireland MG. Development of a selective weakening approach for the seismic retrofit of reinforced concrete structural walls . [M.E.]. Christchurch: Uni. of Canterbury; 2007.
- [5].Kam WY, Pampanin S. Selective weakening techniques for retrofit of existing reinforced concrete structures. *Proc. of 14th World Conf on Earthquake Eng*; 2008 12-17 Oct 2008; Beijing, China, 05-03-0074, 2008.
- [6].Kam WY, Pampanin S. Experimental and numerical validation of selective weakening retrofit for existing non-ductile R.C. frames. *Proc. of ATC-SEI Conf on Improving the Seismic Performance of Existing Buildings and Other Structures*; 2009 9-11 Dec 2009; San Francisco, CA, Paper No. 1585 (S7-2), 2009. p. 706-720.
- [7].Kam WY, Pampanin S, Bull DK. Experimental validation of selective weakening approach for the seismic retrofit of exterior beam-column joints. *Proc. of 2009 NZSEE Conference*; 2009; Christchurch, NZ, Paper No. 32, 2009.
- [8].Kam WY, Pampanin S, Bull DK. Selective weakening retrofit for existing R.C. structures - concept, validation and design example. *Proc. of 9th US National / 10th Canadian Conf on Earthquake Eng*; 2010 25-29th July 2010; Toronto, Canada, Paper No. 395, EERI, 2010.
- [9].Kam WY, Quintana-Gallo P, Akguzel U, Pampanin S. Influence of slab on the seismic response of sub-standard detailed exterior reinforced concrete beam column joints. *Proc. of 9th US National / 10th Canadian Conf on Earthquake Eng*; 2010 25-29th July 2010; Toronto, Canada, Paper No. 922, EERI, 2010.
- [10].NZSEE. New Zealand Engineer Mission to Padang, Indonesia. [Website] 2010 [cited 23rd Jan 2010]; Available from: <http://db.nzsee.org.nz/EqPadang.xml>
- [11].Pampanin S. Vulnerability assessment and retrofit strategies for existing under-designed R.C. frame buildings. *Proc. of NZSEE 2005 Conference*; 2005; Wairakei, New Zealand, 2005.
- [12].Park R. A summary of result of simulated seismic load tests on reinforced concrete beam-column joints, beams and columns with substandard reinforcing details. *J of Earthquake Eng*. 2002;6(2):147-174.
- [13].Wu C-I, Chai J-F, Lin C-CJ, Lin F-R. Reconnaissance report of 0512 China Wenchuan Earthquake on schools, hospitals and residential buildings. *Proc. of 14th World Conf on Earthquake Eng*; 2008 12-17 Oct 2008; Beijing, China, S31-004, 2008.